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EDF-2613, Hydrodynamic and Structural Analyses of Flood Hazards at the PEWE and LET&D Buildings During a Peak Flow in the Big Lost River

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Hydrodynamic and Structural Analysis of Flood Hazards at the PEWE and LET&D Buildings During a Peak Flow in the Big Lost River

Prepared for:
U.S. Department of Energy
Idaho Operations Office
Idaho Falls, Idaho

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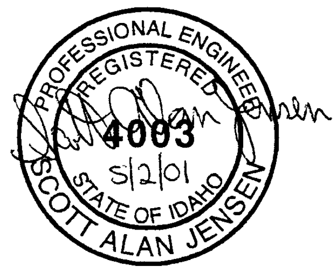
Hydrodynamic and Structural Analyses of Flood Hazards at the PEWE and LET&D Buildings During a Peak Flow in the Big Lost River

The following Engineering Design File (EDF) was prepared under the responsible charge of the Professional Engineer as indicated by the seal and signature provided on this page. The Professional Engineer is registered in the State of Idaho to practice Civil and Structural Engineering.



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1. Project File No.: ____________ 2. Project/Task: INTEC Flood Hazard Analysis

3. Subtask: Hydrodynamic and Structural Analyses of Flood Hazards at INTEC

4. Title: Hydrodynamic and Structural Analyses of Flood Hazards at the PEWE and LET&D Buildings During a Peak Flow in the Big Lost River

5. Summary: This summary briefly describes the problem to be addressed, gives a summary of the analyses performed in addressing the problem, and states the results, conclusions, and recommendations.

A study performed by the INEEL in 1986 estimated the flow volumes and water-surface elevations which would occur during a peak flow in the Big Lost River at the INEEL. This study assumed that the 100-year peak flow and failure of Mackay Dam occur simultaneously, which leads to a conservative estimate of the floodplain. The PEWE (CPP-604, CPP-605) and LET&D (CPP-1618) buildings lie within this flood plain boundary based on the computed water elevation. The purpose of this analysis is to provide information to Idaho DEQ, in order to ensure compliance with RCRA regulations that require determination of hydrodynamic and hydrostatic forces expected to occur at the site and a description of flood protection devices at the facility and how these will prevent washout. The analysis consists of three parts:

(1) A hydrostatic analysis was used to compute the pressure exerted on the building by stationary flood water and saturated soil.
(2) A hydrodynamic analysis was used to compute the pressure exerted on the building by moving flood water caused by wind-generated water waves.
(3) A structural analysis was used to determine whether the concrete foundation of the buildings can withstand the presence of flood water and to assess the extent of water infiltration.

The results of this analysis lead to the following conclusions:

(1) The most important feature of the building construction is whether the first level finished floor elevation is higher than the flood water elevation. Only CPP-1618 meets this requirement, but with a contingency. For CPP-1618, the elevation of wind-generated water waves is higher than the first level floor elevation, and so a barrier is needed to stop waves splashing onto the doorways. For CPP-604 and CPP-605, the flood water elevation is higher than the first level floor elevation, and so additional flood protection devices are needed to prevent water infiltration.
(2) The construction of the buildings follows many of the standard practices used to assure a watertight foundation and to provide adequate drainage during a flood, though some minor water seepage currently occurs through pipe penetrations at CPP-604. Water entering the tank vault and pump pit drains to a sump and is transferred by steam jets to the evaporator feed tank. The jets have enough capacity to transfer the maximum seepage rate expected to occur during a flood.
(3) An important consideration in regard to flood protection is the ability of the retaining walls to withstand lateral earth pressure and water pressure. The exterior retaining walls of the CPP-604 and CPP-605 buildings are considerably stronger with regard to hydrodynamic and hydrostatic forces than are those of the CPP-1618 building. Therefore, for the purposes of this study, information on the strength of the building CPP-1618 retaining wall is provided to represent the minimum wall strength of all three buildings. A structural analysis of the CPP-1618 retaining wall demonstrates that the building can withstand hydrostatic forces caused by the maximum credible flooding event in the Big Lost River on the INEEL.

6. Distribution (complete package): P. E. Murray, MS 3760; N. C. Hutten, MS 3428; S. A. Davies, MS 3650; S. A. Jensen, MS 3650; S. L. Austad, MS 3650

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Introduction

In 1986, the INEEL published a report containing calculated flow volumes and water-surface elevations which would occur during a peak flow in the Big Lost River at the INEEL (1). The INEEL study included the assumption that the 100-year peak flow and failure of Mackay Dam occur simultaneously, and thereby estimated that the peak flow in the Big Lost River is equal to 28,500 ft^3/s at the INEEL diversion dam. However, there are conflicting scientific opinions regarding the magnitude of the 100-year peak flow in the Big Lost River, and the INEEL Natural Phenomena Hazards Committee is currently addressing this issue. Presently, the water surface profile associated with a 28,500 ft^3/s flow is considered to be an upper bound on potential flooding at the INEEL. The particular water surface profile obtained from the INEEL study is used as a basis for the present analysis.

In the INEEL study, 57,740 ft^3/s was estimated to occur at Mackay Dam. The flow is attenuated downstream, and the INEEL diversion dam located in the southwestern part of the INEEL was estimated to receive 28,500 ft^3/s. The diversion dam was assumed to be unable to retain that flow, and so a large part of the discharge flows onto the site. The remaining water was assumed to flow through the diversion channel and into spreading areas. A hydraulic model was used to compute the flow volumes and water elevations within a 18 mile reach downstream of the diversion dam. Several RCRA-regulated buildings lie within the maximum credible flood plain boundary that is based on computed water elevations given in the 1986 INEEL report (1).

The purpose of this engineering analysis is to provide information to Idaho DEQ regarding the hydrodynamic and structural effects of a peak flow. This analysis is performed to ensure compliance with RCRA regulations (2) that require an "engineering analysis to indicate the various hydrodynamic and hydrostatic forces expected to result at the site as a consequence of a 100-year flood," and "structural or other engineering studies showing the design of operational units and flood protection devices at the facility and how these will prevent washout." In the RCRA regulations (2), the term "washout" is defined as "the movement of hazardous waste from the active portion of a facility as a result of flooding."

This analysis is performed to ensure compliance with the following specific requirements stemming from application for a RCRA permit for mixed hazardous waste treatment in the Process Equipment Waste Evaporator (PEWE) and Liquid Effluent Treatment and Disposal (LET&D) facilities:

1. A description of building construction parameters which prevent water infiltration into the units described in the RCRA Volume 14 Part B permit application;
2. A professional engineer (PE) certification that the buildings could withstand hydrodynamic and hydrostatic forces as a result of the flood event described in the 1986 INEEL report (1);
3. PE certification that the design of operational units and/or flood protection devices in the buildings are adequate to prevent washout;
4. A discussion of the controls within the buildings that provide protection against washout.

This analysis consists of three parts:

1. A hydrostatic analysis was used to compute the pressure exerted on the building by stationary flood water and saturated soil;

2. A hydrodynamic analysis was used to compute the pressure exerted on the building by moving flood water generated by a 60 mph wind;
3. A structural analysis was used to determine whether the concrete foundation of the buildings can withstand the presence of flood water and to assess the extent of water infiltration.

Background

Peak Flow Analysis

Koslow and Van Haaften (1) examined the consequences of a failure of Mackay Dam and performed a hydraulic analysis to determine the extent of the flood plain for several scenarios. Their analysis included a predicted 100-year flood and simultaneous piping failure at Mackay Dam, which leads to a breach of the dam, overtopping of the INEEL diversion dam, and flooding of the INEEL site. This scenario results in a peak flow released from the dam that was calculated to be 57,740 ft^3/s. This flow between Mackay Dam and the INEEL is attenuated by storage, agricultural diversion, and channel infiltration. The calculated flow at the INEEL diversion dam is 28,500 ft^3/s. Since the diversion dam is unable to retain the high flow, most of the flood water is assumed to flow onto the site.

Flow Routing Analysis

The peak flow estimated by Koslow and Van Haaften (1) was used in a flow routing analysis to determine the extent of the flood plain at the INEEL site. The geometry of the channel was determined from USGS topographical maps, and the Big Lost River stream bed was examined to determine surface roughness. The Bernoulli equation for ideal flow and the Manning relation for energy loss in open channels were used to compute the peak flow and water elevation at each cross-section. All vertical elevations are in reference to the National Geodetic Vertical Datum of 1929 (NGVD29). Of particular interest in this study are the RCRA-regulated buildings located at the INTEC facility. The leading edge of the flood wave is estimated to arrive at INTEC approximately 17.1 hours after breach of the dam. The peak flow is attenuated to 24,870 ft^3/s, and the peak water velocity is estimated to be 2.2 ft/s. Since the area surrounding INTEC is very flat, flood water will spread easily and so the flood plain is wide and shallow. The elevation of the stream bed in the vicinity of INTEC is 4911 feet and the corresponding calculated water elevation is 4916 feet. The ground elevation at INTEC varies from 4912 ft to 4914 ft. These results suggest that the depth of flood water may reach 4 feet at some locations. Therefore, a maximum water depth equal to 4 feet is used in the following hydrodynamic and hydrostatic analyses.

Koslow and Van Haaften (1) also performed an analysis to examine the potential for overland flooding due to localized heavy rain and snowmelt. It was found that localized flooding due to a 25-year peak rainfall and simultaneous snowmelt lead to a peak flow equal to 32 ft^3/s. This runoff can be accommodated by the drainage basin at INTEC and flood control devices such as culverts, dikes, and ditches. Meanwhile, flood water may collect in low-elevation areas at INTEC. The following hydrodynamic and hydrostatic analyses may also be used to assess the effect of overland flooding due to localized precipitation.

Description of Buildings

The following structures comprise the PEWE and LET&D facilities and have been analyzed for flood hazards:

CPP-604 Process Equipment Waste Evaporator (PEWE) Building

CPP-605 Atmospheric Protection System Fan Building (attached to PEWE)

CPP-1618 Liquid Effluent Treatment and Disposal (LET&D) Building

An important consideration is the first level finished floor elevation, which must be above the flood water elevation in order to prevent infiltration of water through unsealed doorways and other openings. Vertical elevations are currently measured in reference to the National Geodetic Vertical Datum of 1929 (NGVD29). The first level finished floor elevations, as shown on the as-built drawings, are listed in Table 1. The PEWE buildings were constructed in the 1950s, and the datum used then was not NGVD29. Therefore, the INTEC site was recently surveyed by INEEL engineers to determine building and ground elevations using NGVD29. These results are also listed in Table 1. The recent elevation measurements using NGVD29 are approximately one foot less than the elevation shown on the as-built drawings. Note that the PEWE buildings (CPP-604 and CPP-605) are connected and are essentially one building.

Table 1. Building elevations in feet above sea level.

Building	First Level Floor Elevation (shown on as-built drawing)	First Level Floor Elevation (in reference to NGVD29)
CPP-604	4913	4912.0
CPP-605	4913	4912.0
CPP-1618	4917	4916.1

The first level finished floor elevations of CPP-604 and CPP-605 are below the hypothetical flood water elevation. In this case, flood water may infiltrate through exterior doorways and enter the cells. Therefore, additional flood protection devices are needed to prevent washout of hazardous waste in the event of a flood.

Hydrostatic and Hydrodynamic Analyses

In this section, hydrostatic and hydrodynamic analyses will be used to compute the pressure exerted on a building foundation by stationary flood water, saturated soil, and wind-generated water waves.

Hydrostatic Forces

The lateral earth pressure of saturated soil includes the effect of water pressure and soil pressure. Using the method described in Section 2.4 in Peck et al (3), the at-rest earth pressure due to the weight of soil is

$$P_{soil} = K_o(\gamma_{sat} \cdot H - \gamma_{water} \cdot H) = 0.375 \cdot \left(135\frac{lb}{ft^3} - 62.4\frac{lb}{ft^3}\right) \cdot H = 27.2\frac{lb}{ft^3} \cdot H,$$

where P_{soil} is the earth pressure, γ_{sat} is the weight of saturated soil, γ_{water} is the weight of water, H is the soil depth, and K_o is the at-rest earth pressure coefficient. The earth pressure coefficient is obtained from the relation

$$K_o = 1 - \sin\phi,$$

where ϕ is the angle of internal friction which is equal to 43^o according to the INEEL soils report (4). The weight of saturated soil at the INEEL is assumed to be equal to the weight of saturated, dense, mixed-grain sand, as is given in Table 1.4 in Peck et al (3). The resultant force per unit width of retaining wall is

$$F_{soil} = \frac{1}{2} P_{soil} \cdot H = 13.6 \frac{lb}{ft^3} \cdot H^2,$$

where F_{soil} is the resultant force that occurs at a height equal to $H/3$ from the base of the retaining wall, as is shown in Fig. 1. The hydrostatic pressure due to the presence of water is

$$P_{water} = \gamma_{water} \cdot (H + d) = 62.4 \frac{lb}{ft^3} \cdot (H + d),$$

where P_{water} is the hydrostatic pressure, and d is the water depth. The resultant force per unit width of retaining wall is

$$F_{water} = \frac{1}{2} P_{water} \cdot (H + d) = 31.2 \frac{lb}{ft^3} \cdot (H + d)^2,$$

where F_{water} is the resultant force that occurs at a height equal to $(H + d)/3$ from the base of the retaining wall, as is shown in Fig. 1. The total resultant force per unit width of retaining wall is

$$F_{total} = F_{soil} + F_{water} = 44.8 \frac{lb}{ft^3} \cdot H^2 + 31.2 \frac{lb}{ft^3} \left(2 \cdot H \cdot d + d^2\right),$$

where F_{total} is the total resultant force that includes the weight of soil and water.

In the case of saturated soil and a water depth equal to zero, the resultant force per unit width of retaining wall is obtained from the preceding equation by setting $d = 0$:

$$F_{sat\ soil} = 44.8 \frac{lb}{ft^3} \cdot H^2.$$

In the case of dry soil, the resultant force per unit width of retaining wall is

$$F_{dry\ soil} = \frac{1}{2} \cdot K_o \cdot \gamma_{dry} \cdot H^2 = \frac{1}{2} \cdot 0.375 \cdot 118 \frac{lb}{ft^3} \cdot H^2 = 22.1 \frac{lb}{ft^3} \cdot H^2.$$

The density of dry soil is given in the INEEL soils report (4). These results show that a substantial increase in lateral earth pressure occurs when the soil becomes saturated. In fact, the at-rest lateral earth pressure of saturated soil is approximately two times larger than the pressure of dry soil. Since the topmost 40 feet of soil at INTEC is mostly sandy gravel that is dry and permeable (4), the assumption of saturated soil is very conservative. Therefore, the calculated earth pressure is an upper bound on the actual earth pressure that would occur during a flood.

The results of the Koslow and Van Haaften study (1) show that the depth of flood water may reach 4 feet at INTEC during a peak flow in the Big Lost River. Assuming a water depth equal to 4 feet, the total resultant force per unit width of retaining wall is

$$F_{total} = 44.8\frac{lb}{ft^3}\cdot H^2 + 249.6\frac{lb}{ft^2}\cdot H + 499.2\frac{lb}{ft}.$$

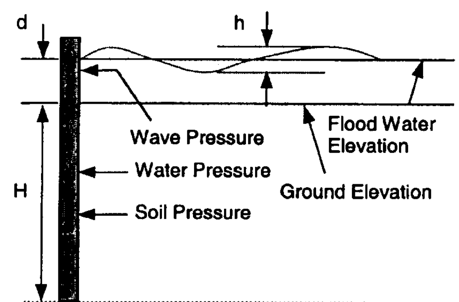

Fig. 1. Various forces acting on a retaining wall during a flood.

Hydrodynamic Forces

The force of moving flood water is calculated by considering the impact of shallow water waves caused by a high wind. A graph that shows the relation between wind velocity, water depth, wave height, and wave period is given in Fig. 10-16 on page 10-36 in Brater and King (5). Assuming a wind velocity equal to 60 mph and a water depth equal to 4 feet, the graph in Brater and King (5) shows that the wave height is 2.0 feet and the wave period is 3.4 seconds. The relation between wave period and wavelength of shallow water waves is

$$\frac{L}{T} = \sqrt{g\cdot d},$$

where L is the wavelength, T is the wave period, d is the water depth, and g is the gravitational acceleration. Assuming a water depth equal to 4 feet and a wave period equal to 3.4 seconds, the wavelength is

$$L = T\sqrt{g\cdot d} = 3.4\,s\sqrt{32.2\frac{ft}{s^2}\cdot 4\,ft} = 38.6\,ft,$$

and the wave velocity is

$$\frac{L}{T}=\frac{38.6\,\text{ft}}{3.4\,\text{s}}=11.35\frac{\text{ft}}{\text{s}}.$$

In comparison, the velocity of flood water as estimated by Koslow and Van Haaften (1) is 2.2 ft/sec. Therefore, the velocity of moving flood water is small in comparison to the velocity of wind-generated waves.

The resultant force per unit width of retaining wall, which is caused by wind-generated waves, is calculated from an empirical relation described on page 10-41 in Brater and King (5). Assuming a wave height equal to 2.0 feet, the pressure exerted by the wave is

$$P_{wave}=\gamma_{water}\cdot h=62.4\frac{\text{lb}}{\text{ft}^3}\cdot 2.0\,\text{ft}=124.8\frac{\text{lb}}{\text{ft}^2},$$

where h is the wave height. According to Fig. 10-21 on page 10-42 in Brater and King (5), the pressure distribution is uniform from the ground to the still-water height, and hydrostatic from the still-water height to a height above still water equal to $1.66\cdot h$. This particular distribution represents the pressure that is produced by a non-breaking wave reflected from a vertical wall. Superposition of approaching and reflecting waves lead to standing waves that have a height approximately equal to $2\,h$. Assuming a water depth equal to 4 feet, the force of the wave is

$$F_{wave}=P_{wave}\left(d+0.5\cdot 1.66\cdot h\right)=124.8\frac{\text{lb}}{\text{ft}^2}\left(4\,\text{ft}+0.5\cdot 1.66\cdot 2.0\,\text{ft}\right)=706.4\frac{\text{lb}}{\text{ft}},$$

and occurs at a height above grade equal to 2.9 feet, as is shown in Fig. 1. The preceding equation for the wave force is the total hydrodynamic force per unit width of retaining wall.

Comparison of Hydrostatic and Hydrodynamic Forces

The hydrostatic and hydrodynamic forces have been calculated using a flood water depth equal to 4 ft and several values of saturated soil depth. The results are shown in Fig. 2.

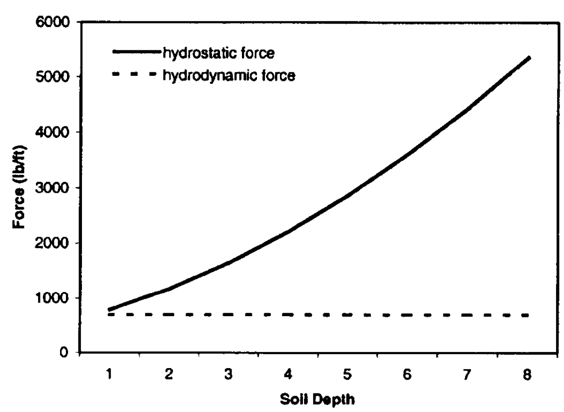

Fig. 2. Hydrostatic and hydrodynamic forces per unit width of foundation.

These results show that for a saturated soil depth larger than several feet, the hydrodynamic force due to waves is small in comparison to the hydrostatic force due to lateral earth pressure and water pressure.

Structural Analysis

In this section, structural analyses will be used to determine whether the concrete foundations of the buildings can withstand the presence of flood water, and to assess the extent of water infiltration through pipe penetrations and exterior doorways.

Foundation Walls

An important consideration in regard to flood protection is the ability of the retaining walls to withstand lateral earth pressure. In the section on hydrostatic analysis, the at-rest lateral earth pressure of saturated soil was computed and shown to be 2 times larger than the pressure of dry soil. This particular flood hazard affects all below-grade retaining walls that support backfill. The structural design of the building foundation is complex, and the concrete retaining walls have a variable height, width, and thickness. Surcharge loads are present in addition to lateral earth pressure. Furthermore, the strength of reinforced concrete depends on the exact size, number, and placement of the steel bars. Therefore, a thorough assessment of the effect of soil saturation on the stress in retaining walls is a complex structural analysis that is beyond the scope of this study. However, the following observations suggest a simple way to assess the strength of the below-grade retaining walls and to demonstrate that the walls are more than adequate to support the increase in lateral earth pressure which may occur as a consequence of a flood.

The buildings comprising the PEWE and LET&D facilities are much different in design. Building CPP-604 includes three levels: one level above grade and two levels below grade. The lower levels contain an inner cell structure surrounded by corridors, storage tank vaults, and various utility rooms. The retaining walls on the lower levels are 1 to 4 feet thick reinforced concrete. Building CPP-605 is attached to CPP-604 and contains a control room and off-gas equipment, but does not have levels below grade. The smallest exterior retaining wall of CPP-605 is 9 in. thick concrete supported by a concrete footing located 5 ft below the first level slab. Building CPP-1618 has three levels that are above grade and none below grade. The exterior retaining wall of CPP-1618 is 8 in. thick concrete supported by a concrete footing located 9 ft below the first level slab. The exterior retaining walls of the CPP-604 and CPP-605 buildings are considerably stronger with regard to hydrodynamic and hydrostatic forces than are those of the CPP-1618 building. Therefore, for the purposes of this study, information on the strength of the building CPP-1618 retaining wall is provided to represent the minimum wall strength for all three buildings. The following structural analysis of a concrete retaining wall uses the design of CPP-1618 to demonstrate that the buildings can withstand hydrostatic forces caused by the maximum credible flood.

The distance from the footing to the first level slab is 7 ½ ft. The first level slab is 3 ½ ft above grade, and so the depth of soil at the base of the retaining wall is 4 ft. Since the flood water elevation coincides with the first level slab elevation, as shown in Table 1, the depth of flood water is 3 ½ ft.

Consider a concrete beam pinned at both ends and acted on by hydrostatic forces, as is shown in Fig. 3. This particular beam loading represents the lateral earth and water pressure acting on a section of retaining wall. Simple supports are assumed since an angular displacement may occur at the ends of the beam where the wall is anchored to the slab and footing. Furthermore, the soil pressure at the inside of the wall is assumed to be equal to the at-rest earth pressure of dry soil.

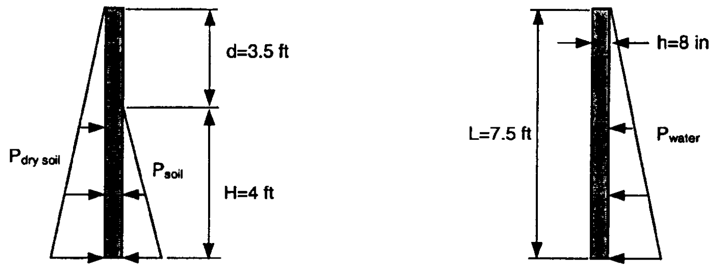

Fig. 3. Lateral earth and water pressure acting on a retaining wall.

The length of the beam is equal to 7 ½ ft and the thickness of the beam is equal to 8 in. Using the hydrostatic pressure calculated previously, the pressure at the base of the beam is equal to

$$P = P_{soil} + P_{water} - P_{dry\ soil} = 27.2\frac{lb}{ft^3}\cdot H + 62.4\frac{lb}{ft^3}\cdot(H+d) - 44.3\frac{lb}{ft^3}\cdot(H+d) = 245\,lb/ft^2,$$

where $H = 4$ ft and $d = 3.5$ ft. To examine the loading on the beam, assume that the pressure varies linearly from zero to 245 lb/ft^2 over the length of the beam, as shown in Fig. 4.

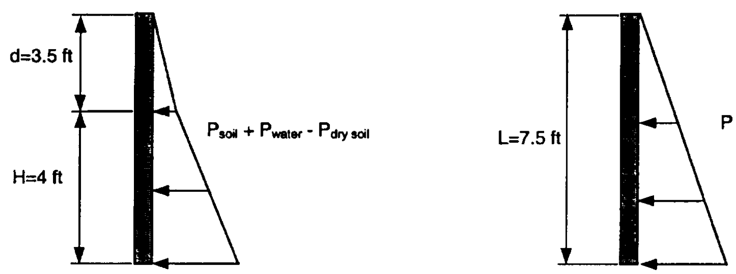

Fig. 4. Actual and assumed pressure acting on a retaining wall.

The maximum shear force and bending moment occur at the base of the beam, and are obtained from the following formulas found in Roark and Young (6), Table 3, Case 2e:

$$M = 0.0641\,P\,L^2 = 883\frac{ft-lbs}{ft},$$

$$V = \frac{P\,L}{3} = 613\,\frac{\text{lbs}}{\text{ft}}.$$

The actual force and moment are multiplied by a load factor equal to 1.7, as specified in ACI 318 (7), to give $M_u = 1501\ \text{lb ft}$ and $V_u = 1042\ \text{lb}$ per 1 foot width of beam.

To compute the allowable shear and moment capacity of the concrete beam, assume that the beam includes vertical reinforcement only and neglect the presence of horizontal reinforcement. The vertical reinforcement is a single layer of #4 bar spaced 10 inches center to center. This meets the requirement that the area of vertical reinforcement shall not be less than 0.0015 times the wall area, as described in Sections 14.2.11 of ACI 318 (7), which was the building code for reinforced concrete at the time CPP-1618 was built. Since the bar is placed in the center of the slab, the top and bottom covers are equal to 4 inches. The concrete has a minimum compressive strength equal to 3000 psi. Furthermore, it is assumed that the yield strength of reinforcement bar is equal to 40,000 psi.

The computation of moment and shear capacity are based on ACI 318 (7) and the CRSI Design Handbook (8). The shear capacity is obtained from Section 11.3.1.1 of ACI 318 (7):

$$V_c = 0.85\,2\sqrt{f_c'}\,b\,d,$$

where f_c' is the compressive strength of concrete, b is the width of the beam, and d is the distance from the extreme compression fiber to the center of mass of the tension reinforcement. The moment capacity for a single layer of tension reinforcement is obtained from page 5-7 in the CRSI Design Handbook (8):

$$M_n = 0.90\,A_s\,f_y\,(d - a/2),$$

where A_s is the area of tension reinforcement, f_y is the yield strength of the reinforcement, and a is the depth of the concrete compression block which is obtained from a balance of concrete compression and bar tension:

$$A_s\,f_y = 0.85\,f_c'\,b\,a.$$

The moment capacity of the concrete beam is $M_n = 2767\ \text{lb ft}$ per 1 foot width of beam, which exceeds the factored moment computed above. The shear capacity of the beam is $V_c = 4470\ \text{lb}$ per 1 foot width of beam, which exceeds the factored shear computed above. In fact, the retaining walls at CPP-1618 are stronger than this simple example indicates, owing to the presence of intersecting walls and columns anchored to each section of retaining wall, and the presence of horizontal reinforcement.

Water Infiltration Through Pipe Penetrations

The waste treatment process at INTEC includes a complex system of pipes that transfer waste between buildings for treatment. This observation suggests the potential for seepage caused by water infiltration through pipe penetrations located below the flood level. A field investigation found that seepage currently occurs through pipe penetrations into the waste tank vault and waste pump pit located at CPP-604. Seepage occurs only at pipes entering the tank vault and pump pit through penetrations located at the roof of the concrete enclosure. As a result of rain, snowmelt, and runoff, the soil above the concrete enclosure becomes wet and seepage occurs. Seeping water is currently a minor nuisance, but the potential exists for additional infiltration into CPP-604 during a flood. This infiltration will be handled by sumps and jets designed to route water to storage tanks that are equipped with level monitors and overflow alarms. It is important to note that seepage does not penetrate the inner cell structure nor penetrate the pipes carrying waste. A waste pipe is encased in a larger pipe that is well sealed, which produces a secondary containment that keeps the seepage and waste stream separated.

Seepage into CPP-604 has been recently monitored by tracking the amount of water transferred to the evaporator feed tank. The total volume of water includes sump water, process water, and steam condensation. By subtracting the process water and steam condensation from the total, the amount of water transferred from the sumps had been calculated. However, the sump water had included leakage from steam valves. In November 2000, the steam system was turned off between transfers to prevent steam leakage. Since then, data on water seepage has been available because the sump receives only the water seepage through pipe penetrations.

The tank vault consists of four separate concrete enclosures. The penetrations into the enclosure containing tank VES-WM-100 leak, and the penetrations into the enclosure containing tanks VES-WL-101 and VES-WL-102 leak. The penetrations into the enclosure containing the pumps also leak. The pump pit and each tank enclosure have a sump and a steam jet. Water infiltrating the tank vault and pump pit travels directly to the sump, where the jets are used to transfer the sump water to VES-WL-132 and VES-WL-150, and then to VES-WL-133 (the evaporator feed tank). The seepage for each month from November 2000 through February 2001 is shown in Fig. 5.

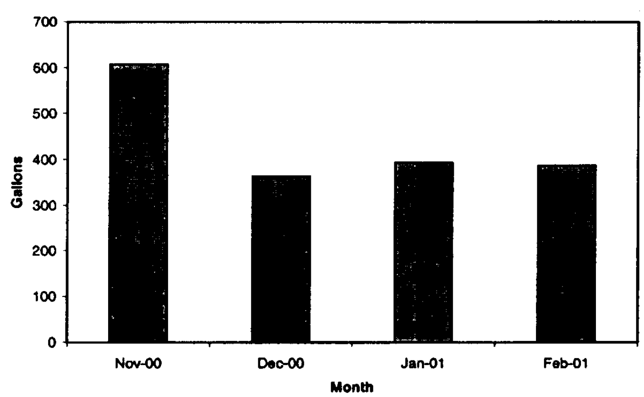

Fig. 5. Seepage through pipe penetrations in CPP-604.

The concrete enclosures are buried under an earth embankment that is at the same elevation as the first floor roof of CPP-604. The difference in elevation between the pipe penetrations and the top of the embankment is 32 ft. The difference in elevation between the ground level and the top of the embankment is 14 ft. Therefore, the difference in elevation between the pipe penetrations and the ground level is 18 ft.

The maximum seepage rate expected to occur during the hypothetical flood is estimated in the following manner. Note that the maximum seepage occurred in November and was equal to 20 gal/day or 0.833 gal/hr. Assume that the hydraulic head causing this seepage is equal to 18 ft, which is the difference in elevation between the pipe penetrations and the ground level. Since the embankment has been observed to be wet during the winter and spring seasons, it is plausible that part of the embankment is saturated. Furthermore, it is assumed that the embankment is totally saturated during a flood, and so in that case the hydraulic head is equal to 32 ft.

Since the hydraulic pressure in soil is proportional to the hydraulic head, the hydraulic pressure increases by a factor equal to

$$\frac{h_{max}}{h} = \frac{32}{18} = 1.8 .$$

Note that this is a conservative estimate of the increase in hydraulic pressure because it is assumed that the embankment is totally saturated with water. Since the seepage rate is proportional to the hydraulic pressure, the maximum seepage expected during a flood is

$$Q = 0.833 \frac{gal}{hr} \cdot 1.8 = 1.5 \frac{gal}{hr} .$$

The capacity of a steam jet depends on several factors that include steam pressure, pipe size, suction lift, and discharge head. The jets in the tank vault have a 1 in. inlet line, 1½ in. suction line, and 1½ in. discharge line. The jet in the pump pit has a ¾ in. inlet line, 1 in. suction line, and 1 in. discharge line. To calculate the pump capacity, consider the small jet in the pump pit, which is a Penberthy jet model GL-1 or equivalent. The upper bounds on the lift and head needed to transfer water from the sump to the evaporator feed tank are a suction lift equal to 5 feet and a discharge head equal to 20 feet. The minimum operating steam pressure needed to operate at this suction lift and discharge head is equal to 80 psig at a suction water temperature of 80°F. The steam pressure can be adjusted to increase the flow rate if needed. According to the Penberthy technical data (9), model GL-1 at these conditions has a discharge capacity equal to 7.9 gal/min. Since the small jet can transfer much more than the calculated maximum seepage, the flood protection devices have enough capacity to handle the maximum seepage that is expected to occur at CPP-604 during a flood.

Water Infiltration Through Doorways

In most cases, openings such as doors, stairs, and elevators on the first level lead to the inner cell structure. Since these openings are not watertight, it is necessary that the first level finished floor elevation be located above the flood water elevation in order to prevent infiltration of water through unsealed doorways and other openings. Generally, flood water will not enter the building if the first level is at an elevation at least equal to 4917 ft, which is the flood water level (4916 ft) plus the wave height (1 ft). The data in Table 1 show that neither building meets this requirement. Since the first level finished floor elevation of CPP-1618 is 4916 ft, infiltration of water can be prevented by using a barrier to stop waves splashing onto the doorways. For the other buildings, additional flood protection devices are needed to prevent water infiltration.

Conclusions

An engineering analysis was used to calculate the various hydrodynamic and hydrostatic forces expected to result at the PEWE and LET&D buildings as a consequence of a 100 year flood coinciding with a failure of Mackay Dam. A structural study was used to describe the design of these buildings and their flood protection devices and how these will prevent washout of hazardous waste. Specific details are given below.

The following structural features of the buildings comprising the PEWE and LET&D facilities were examined: footing and foundation structures; openings such as doorways that enable water to easily infiltrate; and the occurrence of water seepage through pipe penetrations. The following results were obtained:

(1) The most important feature of the building construction is whether the first level finished floor elevation is higher than the flood water elevation. Only CPP-1618 meets this requirement, but with a contingency. For CPP-1618, the elevation of wind-generated water waves is higher than the first level floor elevation, and so a barrier is needed to stop waves splashing onto the doorways. For CPP-604 and CPP-605, the flood water elevation is higher than the first level floor elevation, and so additional flood protection devices are needed to prevent water infiltration.

(2) The construction of the buildings follows many of the standard practices used to assure a watertight foundation and to provide adequate drainage during a flood, though some minor water seepage currently occurs through pipe penetrations at CPP-604. Water entering the tank vault and pump pit drains to a sump and is transferred by steam jets to the evaporator feed tank. The jets have enough capacity to transfer the maximum seepage rate expected to occur during a flood.

(3) An important consideration in regard to flood protection is the ability of the retaining walls to withstand lateral earth pressure and water pressure. For the purposes of this study, the exterior retaining wall of CPP-1618 was chosen to represent the minimum wall strength of the three buildings evaluated, with regard to withstanding hydrostatic forces acting on a foundation wall. A structural analysis of the CPP-1618 retaining wall demonstrates that the building can withstand hydrostatic forces caused by a maximum credible flooding event.

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